Effects of slab panel vertical support on tensile membrane action

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ABSTRACT: A recently-developed design method predicts composite slab capacities in fire, incorporating the effects of tensile membrane action. The method designs rectangular slab panels including unprotected beams within the panels, and are supported on edges that resist vertical deflection. In practice, vertical support is achieved by protecting the perimeter beams. Generic fire protection ensures that beam temperatures stay below critical temperatures of 550°C or 620°C within the required fire resistance time. However, large vertical displacements of the protected edge beams may cause a loss of the membrane mechanism, inducing single-curvature bending, which may lead to a catastrophic failure of the structure. A finite element investigation into the provision of adequate vertical support along slab panel boundaries has been conducted. The study has examined various degrees of protection relative to the development of the membrane mechanism. Comparisons are made with the membrane action design method and various acceptance criteria.

1 INTRODUCTION

Developments in structural fire engineering over the past 20 years have led to an improved understanding of structural behaviour under fire conditions. More recently, the influence of tensile membrane action in sustaining steel-framed buildings with composite floors, well beyond their traditional failure times, has emphasised the need to incorporate performance-based approaches into the design of structures in fire. Tensile membrane action is a mechanism producing increased load-bearing capacity in thin slabs undergoing large vertical displacements, in which stretched central areas of the slab induce an equilibrating peripheral ring of compression. This large-deflection mechanism relies on two-way bending and the availability of vertical support along the edges of the slab, and occurs with or without horizontal restraint along the edges of the slab. Use of the mechanism in the design of composite floors in fire may help to reduce building costs, as a large number of floor beams can often be left unprotected (Bailey 2004). Also, repair of fire-damaged structures may only involve replacing a few compartments, instead of rebuilding the entire structure.

In the design of composite slabs for tensile membrane action, a floor plate is divided into several rectangular slab panels of low aspect ratio. The beams on the perimeter of each slab panel are protected to provide vertical support, while those in the interior region are left unprotected, as shown in Figure 1. On exposure to fire, the unprotected beams lose strength, and their loads are progressively borne by the slab, under large vertical deflections. The slab's capacity increases as its vertical deflections increase.

Tensile membrane action can be modelled effectively by sophisticated finite element software, such as *Vulcan* (Huang et al. 2002, 2003a, b). Finite element processes are time-consuming, and simpler performance-based methods, such as the Building Research Establishment's membrane action method, are preferred alternatives for routine design.

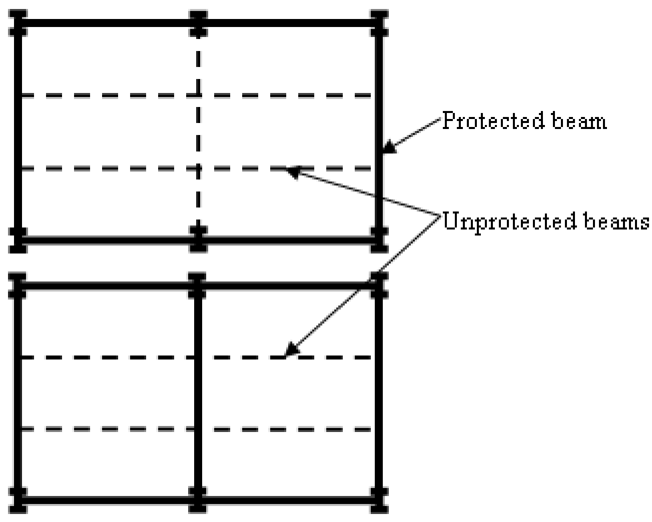

Figure 1: Typical rectangular slab panels for tensile membrane action.

2 BRE MEMRANE ACTION METHOD

The design method developed by Bailey is based on rigid-plastic theory subject to large change of geometry. It calculates the enhancement that tensile membrane action adds to the traditional (Yield-Line) flexural capacity of the slab. The membrane action method divides a floor plate into several rectangular slab panels supported on edges that primarily resist vertical deflection, and which incorporate unprotected composite beams (Bailey 2004) as shown in Fig. 1. The panels are horizontally unrestrained; the unprotected beams and slabs are simply supported, with their capacities determined by lower-bound limit analyses (Bailey & Moore 2000). The method conservatively ignores any contribution of the tensile strength of concrete to the capacity of the slab, and does not provide any information on the state of the protected boundary beams apart from the need to keep them vertically supported. Failure is based on the formation of a full-depth crack across the shorter span of the slab (Bailey 2001).

2.1 *Slab panel capacity*

After a floor slab has been divided into a number of panels, and the required fire resistance of the structure has been established, the residual resistance of each panel is calculated by considering the dimensions of the panel, the resistance of the internal beams and the reinforcement mesh size. This is then compared with the applied load at various times within the required design time. If the residual capacity of the panel is found to be less than the applied load at the fire limit state, then either the capacity of the internal beams or the reinforcement mesh size is increased (Newman et al. 2006).

At first, all floor loads are carried by the unprotected composite beams. As these lose strength, large vertical displacements are induced, which generate self-equilibrating in-plane tensile and compressive forces capable of producing enhancements to the theoretical yield-line capacity of the slab. The method, initially developed for isotropic reinforcement (Bailey & Moore 2000, Bailey 2001), has been extended to include orthotropic reinforcement (Bailey 2003). Recently, the change of in-plane stress distributions and compressive failure have been added (Bailey & Toh 2006).

To facilitate use of the tensile membrane mechanism and the BRE method, a design guide known as P-288 (Newman et al. 2006) has been produced by the Steel Construction Institute. This provides tables of slab panel arrangements and reinforcement sizes that should be used, based on the type of concrete, the geometry of the slab cross-section and the load ratio of the secondary beams. It also provides information on the use of free software (TSLAB) for quick analyses of these slab panels.

2.2 *Panel vertical support*

As vertical support along the slab panel boundaries is realised in practice with edge beam protection, the assumption of continuous vertical restraint at all times during the fire cannot hold true. Numerical analyses have shown that the protected perimeter beams lose strength and stiffness, at some point, allowing the formation of a single-curvature slab-bending mechanism, which will pull on its connections, leading to a catastrophic failure of the structure if these connections are not adequately designed against such forces. This potential failure type has led to the series of finite element studies reported here, into the adequacy of vertical support along the slab panel boundaries.

3 FINITE ELEMENT ANALYSES

The finite element analyses were performed with *Vulcan* (Huang et al. 2000, 2003a, b), a geometrically nonlinear finite element program that includes the effects of nonlinear material behaviour at elevated temperatures. The program models reinforced concrete slabs with 9-noded nonlinear layered rectangular elements, which represent temperature distributions through the depth of the slab by assigning different, but uniform, temperatures to each layer of the element. The slab element can adequately represent bending and membrane action at large displacements. Reinforcement is smeared across the area of the slab element. The program uses a biaxial failure surface for concrete, and so it can adequately represent failure in tension or compression. Beams are modelled with 3-noded nonlinear beam elements.

3.1 *Loading*

Using the SCI P-288 document, a slab panel of dimensions 9.0m x 7.5m (Figure 2) was selected, to be designed for a 60-minute fire resistance. The panel had two unprotected beams spanning in the short direction. Following the guidance in the document for office-type buildings, the unprotected beams could be loaded to their full capacity, and an A193 mesh was sufficient if the slab profile of Figure 3 was used, with normal weight concrete and the loads listed in Table 1.

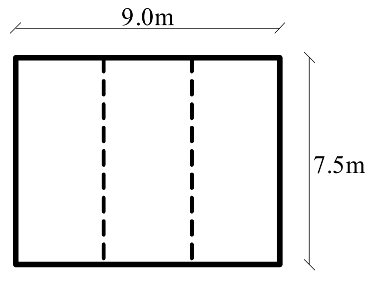

Figure 2: Slab panel geometry.

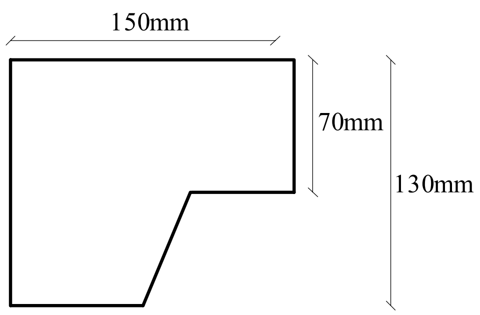

Figure 3: Concrete slab cross-section profile.

Table 1: Design Loading

Permanent Load	kN/m^2
Slab selfweight	2.40
Beam selfweight	0.20
Mesh (A193)	0.03
Imposed Load	
Variable load	3.5
Ceilings/ Services	1.7

Ambient and elevated-temperature design of the floor beams, using BS5950-3 and BS5950-8, resulted in 356x127x33UB as secondary beams and 533x210x82UB as primary beams (Fig. 4).

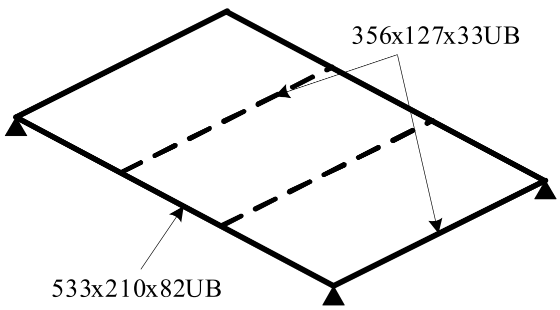

Figure 4: Slab panel, showing protected and unprotected beams.

The beam design factor the unprotected secondary beams was 0.74 (a load ratio of 0.44 in fire). The critical temperatures of the protected secondary and primary beams were 631°C and 646°C respectively. A generic protection scheme was adopted with lightweight fire resisting gypsum boards (density = $800kg/m^3$, specific heat capacity = 1700Jkg/K, conductivity = 0.2W/mK), so that their temperatures were limited to 550°C at 60 minutes.

3.2 *Thermal analysis*

An average slab thickness of 100mm (100mm = (70mm+130mm)/2) was chosen for the structural analysis. A thermal analysis was performed to ascertain the temperature distributions through the unprotected beams and the concrete slab. Temperature distributions calculated to the simplified process from Eurocode 3 Part 1.2 (CEN 2005) were used for the beams, while a one-dimensional thermal analysis was performed to generate the temperature distributions across 13 layers of concrete and reinforcement, using a thermal analysis program (FPRCBC-T), developed by Huang et al. (1996). A comparison of the temperature distributions in the *Vulcan* model and TSLAB, for a 90-minute exposure to the standard temperature-time curve is shown in Figure 5.

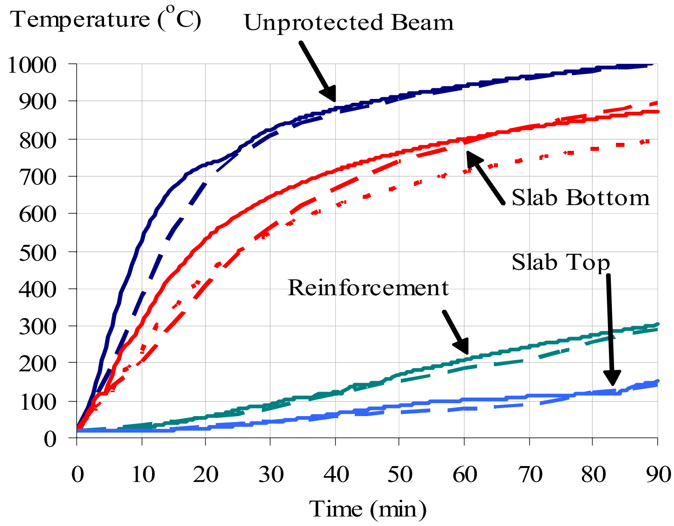

Figure 5: Unprotected beam and slab bottom surface, top and reinforcement temperatures of the slab panel system

TSLAB results are shown as dashed lines while the *Vulcan* temperature distributions are shown as continuous lines. For the bottom of the slab, the dotted line shows the average temperature of the lowest layer of the *Vulcan* slab element while the solid line shows the temperature of the bottom surface. It is observed from the figure above that there was generally good correlation between the thermal distributions in the *Vulcan* and TSLAB models.

3.3 *Structural analyses*

The primary *Vulcan* model was a horizontally-unrestrained slab panel, with vertical restraint only available at the corners (Fig. 4), providing vertical edge support by using protected beams around the perimeter. Other variations of this model (see Table 2) were investigated to evaluate the influence of several parameters on tensile membrane action.

Table 2: *Vulcan* Analyses and Parameters

Condition	*Vulcan* Analyses								
	1	2	3	4	5	6	7	8	9
Generic Protection	✓	✓	✓			✓	✓	✓	
2X Generic Protection									✓
Cold Perimeter Beams				✓	✓				
Corner Vertical Restraint	✓	✓	✓	✓	✓	✓	✓	✓	✓
Edge Vertical Restraint		✓		✓	✓				
Concrete Tensile Strength ignored					✓				
Rotational Restraint on 2 edges								✓	
Rotational Restraint on 3 edges						✓	✓		
Rotational Restraint on 4 edges			✓						

The *Vulcan* analyses were compared with the TSLAB limiting deflection curve, the BRE allowable vertical deflection limit (for fire resistance times up to 90 minutes), the required vertical deflection (from the generic BRE approach) and a limiting deflection of span/20 (7500mm/20 = 375mm). Figure 6 shows the vertical displacements used in the comparison. The black continuous line shows the required vertical displacements obtained by the generic BRE Method with an A193 mesh.

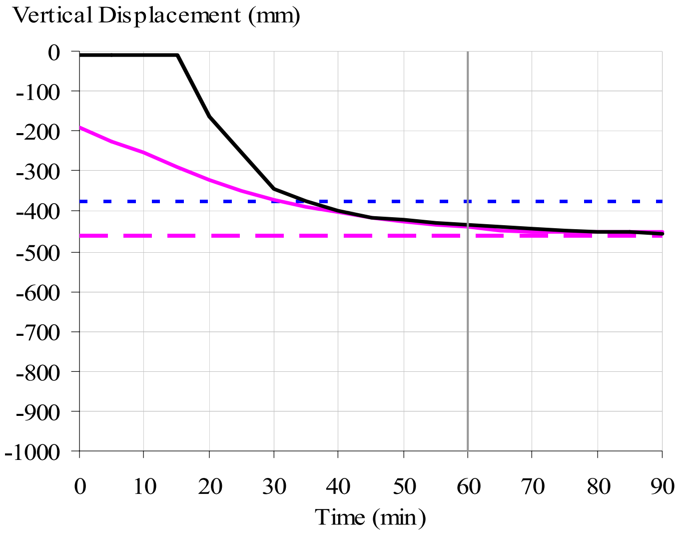

Figure 6: Absolute maximum vertical displacements of the slab panel model given by the BRE Method.

The faint continuous curve shows the limiting vertical displacements generated by the TSLAB program, obtained from Equation 1 below, at each time step with T_2 and T_1 as the bottom and top slab temperatures respectively:

$$V = \frac{\alpha(T_2 - T_1)l^2}{19.2h} + \sqrt{\left(\frac{0.5 f_y}{E}\right)_{\mathrm{Reinf}\ 20^\circ C} \times \frac{3L^2}{8}} \qquad (1)$$

in which α is the coefficient of thermal expansion; l is the length of the shorter side of the slab panel; L is the length of the longer side of the slab panel; h is the thickness of the concrete slab; f_y is the yield stress of the steel reinforcement; and E is the Young's modulus of the reinforcement.

The faint dashed line in Fig. 6 is the allowable absolute vertical deflection limit, as defined in the BRE method. The value of this deflection limit is obtained when $T_2 - T_1$ = 770°C in Equation 1. The line with short dashes represents the deflection at span/20 (375mm). A faint line has been placed on each of the graphs to show that the slab panels were designed for 60 minutes fire resistance.

Figure 6 shows the adequacy of the A193 mesh for the chosen slab panel size and the design criteria of the simplified design method.

4 RESULTS & DISCUSSION

The results of the various *Vulcan* analyses are presented in Figures 7-9 and 11-12. It should be noted that, unless otherwise stated, the results presented in this section all show absolute maximum vertical displacements of the middle of the slab panel system.

4.1 *Vertical restraint*

Figure 7 shows plots from *Vulcan* analyses which were the same except for V2 being supported vertically along its edges. In contrast, V1 suffered collapse of the protected secondary beams, caused by their loss of strength and stiffness as they approached the design temperature of 550°C. This is emphasized in Figure 8 which shows vertical displacements of the middle of the slab panel relative to the vertical displacements of the mid-span of the protected primary and secondary beams.

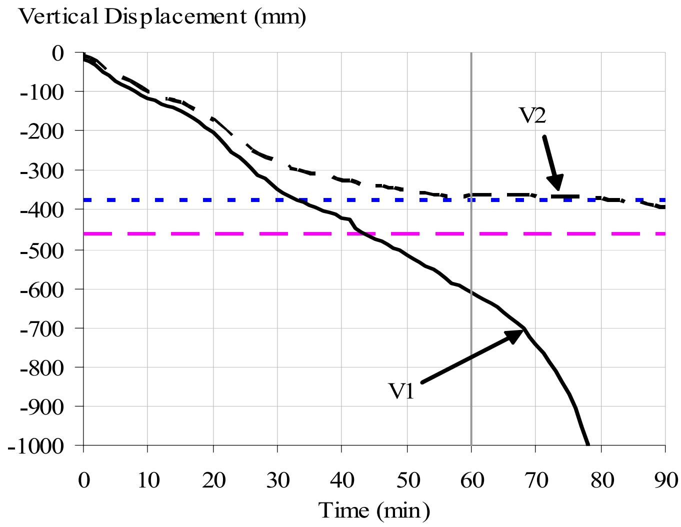

Figure 7: Absolute vertical displacements from *Vulcan* analyses with generic protection and vertical support at corners and along the slab panel edges.

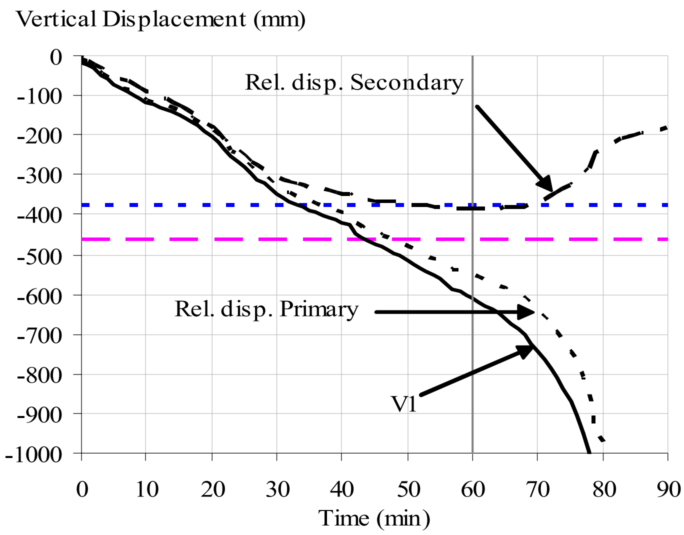

Figure 8: Absolute and relative vertical displacements of *Vulcan* analysis with generic protection and vertical restraints at corners only.

From this figure it is observed that, as the secondary beams reached 550°C, they began to fold, losing the tensile membrane mechanism and allowing the formation of a single-curvature mechanism that produced excessively large displacements. The reduction in difference between the deflections of the centre of the slab and the mid-span of the protected secondary beams shows how rapidly they deflected while the primary beams effectively stayed vertical.

4.2 *Influence of the ensile strength of concrete*

The results (Figure 9) show that neglecting the tensile strength of concrete requires higher vertical deflections to generate the required slab capacity. The *Vulcan* model which included the tensile strength of concrete compared very well with the generic BRE required deflection, and just crosses the allowable vertical displacement limit at about 55 minutes. Part of the large deflections recorded here was due to the buckling of the slab, caused by its expansion against the cold perimeter beams.

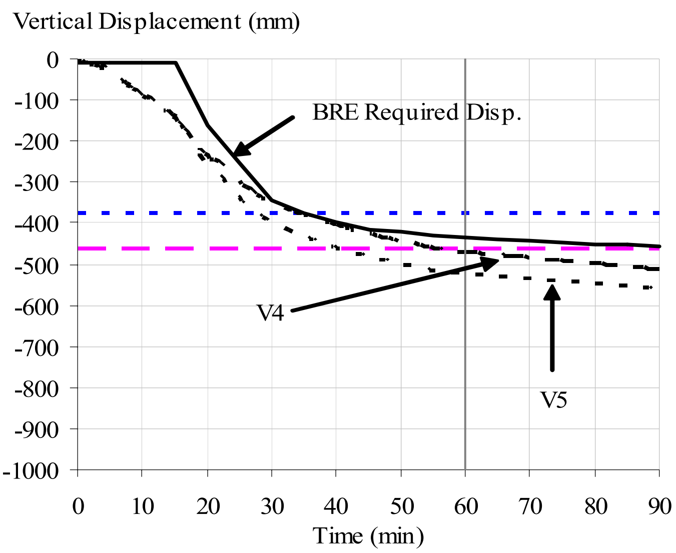

Figure 9: Absolute vertical displacements of *Vulcan* analyses with cold perimeter beams (20°C) and vertical support along all slab panel edges.

4.3 *Effect of continuity at the panel boundary*

The effect of continuity of the slab panels was also investigated. The analyses examined 4 scenarios: one with continuity on 2 adjacent sides, two with continuity on 3 sides and one with continuity on all 4 sides. The analyses were performed on the typical floor layout shown in Figure 10. Figure 11 shows the absolute vertical displacements of the centre of each of the slab panels in Figure 10.

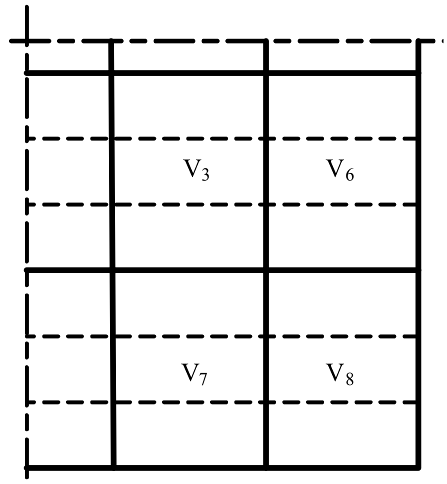

Figure 10: Layout of continuous slab panels in a typical floor plate. Labels correspond to analyses shown in Figure 11.

The results confirm the dependence of the survival of each slab panel on the stiffness of the protected beams, the positive influence adjacent slabs have on the stiffness of the fire-exposed slab.

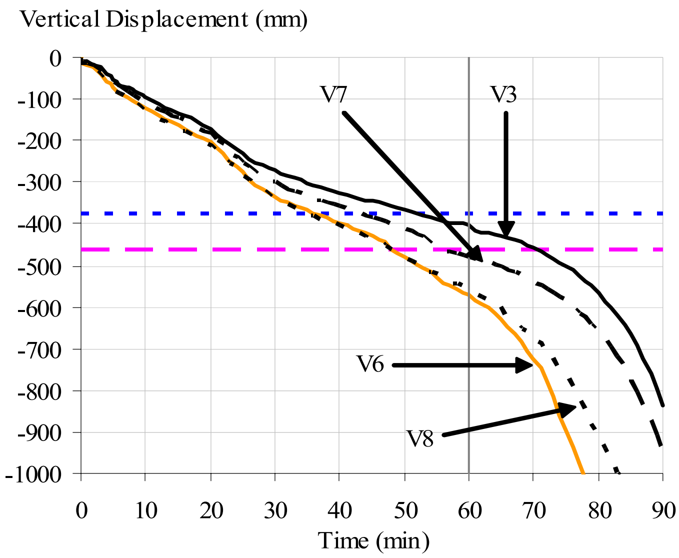

Figure 11: Absolute vertical displacements of *Vulcan* analyses with generic protection, vertical support at corners and rotational restraints along the slab panel edges

4.4 *Thickness of protection material*

A final analysis of Type V1 was conducted. This time, twice the thickness of fire protection was applied to the perimeter beams. Figure 12 shows that the absolute vertical displacements were within the required TSLAB limit. This suggests that the requirement for vertical restraint at the boundaries of the slab panel may require the use of heavier sections or thicker protection, which may have the effect of increasing construction costs.

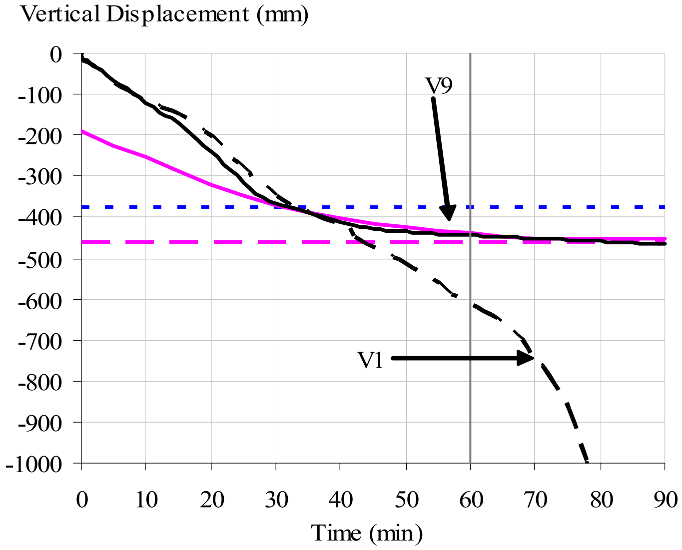

Figure 11: Maximum absolute vertical displacements of *Vulcan* analyses with 2 different thicknesses of protection

5 CONCLUSION

The analyses have examined three protection schemes, and several support conditions. The results suggest that, on condition that the slab panel edges stay vertical throughout the fire, the BRE simplified method and its failure criteria provide good estimates of composite slab behaviour in tensile membrane action. This degree of vertical support is, however, not practical as protected perimeter beams deflect, even at ambient temperature.

In practice, the protected beams would have been designed against attaining their critical temperatures of 631°C and 646°C. The analyses have shown that, even for a design temperature of 550°C, the protected secondary beams lost their necessary degree of support, although they had been designed for the additional loads to be expected due to tensile membrane action.

Restraint from adjacent slabs is clearly beneficial, but for slab panels verging on the façade of a building, increasing the level of protection seems a viable option, although this could potentially be an expensive thing to do.

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